CHAPTER 5. CALCULATION OF RUNOFF

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1.0 OVERVIEW

Calculation of runoff is a critical step in the design of urban storm water management facilities and conveyances. Runoff must be estimated accurately with a reasonable margin of safety to ensure protection of public health, safety and welfare, cost-effective designs, and protection of waterways and natural streams. Three acceptable methods of calculating runoff are presented in this chapter:

1. **Rational Method:** Acceptable for drainage areas less than 100 acres when only peak flow rates are needed. However, the City of Springfield (City) reserves the right to require the use of other methods (i.e., Kinematic Wave or Soil Conservation Service [SCS] Methods, discussed below) for drainage areas less than 100 acres, particularly in cases where land use in the watershed is non-homogeneous or areas of storage exist in the watershed. The SF-Rational Spreadsheet is provided with this manual to assist in peak runoff prediction using the Rational Method.

2. **Kinematic Wave Method:** Acceptable for any size watershed and appropriate for complex watersheds and any drainage areas greater than 100 acres. This method shall be used with the Huff temporal rainfall distribution. The Kinematic Wave Method is the preferred method for most complex urban runoff conditions.

3. **SCS Unit Hydrograph Method:** Acceptable for any size watershed and appropriate for complex watersheds and any drainage areas greater than 100 acres. This method shall be used with the Huff temporal rainfall distribution.

Hydrologic evaluation goes beyond the application of models to calculate runoff peak flow rates and volumes. As a part of hydrologic analysis, a reasonableness check should be conducted on computed peak flow rates and runoff volumes. Reasonableness checks may include methods of calculation such as the Rational Method, other hydrograph methods, or the United States Geological Survey (USGS) regression equations presented as an alternative method in Section 5.0. Other checks may include comparison of actual flood data or other flood studies that may have been conducted by the Federal Emergency Management Agency (FEMA), the U.S. Army Corps of Engineers (USACE), or the City.

2.0 PRECIPITATION

For any of the acceptable calculation methods, it is necessary to define depth-duration-frequency relationships of rainfall. Precipitation data from the *Rainfall Frequency Atlas of the Midwest* (Huff and Angel 1992) shall be used for all runoff computations, as summarized in Table RO-1. For event durations that fall between values listed in the table, the user should use linear interpolation to determine the appropriate depth of precipitation.

2.1 Intensity-Duration-Frequency for the Rational Method

For Rational Method analysis, rainfall intensity in inches per hour must be determined from an event with a duration equivalent to the time of concentration. Tables RO-1 and RO-2 provide depth and intensity values for various rainfall durations and frequencies in the Springfield area.

Table RO-1
Rainfall Depth-Duration-Frequency Relationships from Rainfall Frequency Atlas of the Midwest (Huff and Angel 1992)

Duration	Depth of Precipitation (in)						
	1-year	2-year	5-year	10-year	25-year	50-year	100-year
5 min	0.36	0.45	0.57	0.67	0.79	0.88	0.98
10 min	0.63	0.79	1.01	1.17	1.38	1.54	1.72
15 min	0.81	1.02	1.29	1.50	1.77	1.98	2.21
30 min	1.11	1.39	1.77	2.05	2.43	2.72	3.03
1 hr	1.41	1.77	2.25	2.61	3.08	3.45	3.84
2 hr	1.74	2.19	2.78	3.22	3.80	4.26	4.74
3 hr	1.92	2.41	3.07	3.55	4.20	4.70	5.24
6 hr	2.25	2.83	3.59	4.16	4.92	5.51	6.14
12 hr	2.61	3.28	4.17	4.83	5.71	6.39	7.12
18 hr	2.82	3.54	4.50	5.22	6.17	6.90	7.69
24 hr	3.00	3.77	4.79	5.55	6.56	7.34	8.18
48 hr	3.30	4.14	5.25	6.07	7.17	8.05	8.97
72 hr	3.68	4.62	5.81	6.69	7.90	8.85	9.85
120 hr	4.16	5.21	6.50	7.45	8.70	9.68	10.77
240 hr	5.37	6.59	8.05	9.13	10.49	11.52	12.61

Table RO-2
Rainfall Intensity-Duration-Frequency Relationships from Rainfall Frequency Atlas of the Midwest (Huff and Angel 1992)

Duration	Intensity of Precipitation (in/hr)						
	1-year	2-year	5-year	10-year	25-year	50-year	100-year
5 min	4.32	5.40	6.84	8.04	9.48	10.56	11.76
10 min	3.78	4.74	6.06	7.02	8.28	9.24	10.32
15 min	3.24	4.08	5.16	6.00	7.08	7.92	8.84
30 min	2.22	2.78	3.54	4.10	4.86	5.44	6.06
1 hr	1.41	1.77	2.25	2.61	3.08	3.45	3.84
2 hr	0.87	1.10	1.39	1.61	1.90	2.13	2.37
3 hr	0.64	0.80	1.02	1.18	1.40	1.57	1.75
6 hr	0.38	0.47	0.60	0.69	0.82	0.92	1.02
12 hr	0.22	0.27	0.35	0.40	0.48	0.53	0.59
18 hr	0.16	0.20	0.25	0.29	0.34	0.38	0.43
24 hr	0.13	0.16	0.20	0.23	0.27	0.31	0.34
48 hr	0.07	0.09	0.11	0.13	0.15	0.17	0.19
72 hr	0.05	0.06	0.08	0.09	0.11	0.12	0.14
120 hr	0.03	0.04	0.05	0.06	0.07	0.08	0.09
240 hr	0.02	0.03	0.03	0.04	0.04	0.05	0.05

2.2 Huff Temporal Rainfall Distribution

When using the Kinematic Wave Method or SCS Method for runoff computations, the Huff rainfall distribution shall be used for the temporal distribution. The Huff distribution is presented in Table RO-3 and is expressed as cumulative percentages of total duration and total rainfall accumulation. Different families of Huff distribution curves are applicable for different drainage areas. Table RO-3 is applicable to drainage areas less than 10 square miles. For larger drainage areas, refer to the *Rainfall Frequency Atlas of the Midwest* (Huff and Angel 1992). Each family of curves consists of four storms (first-quartile, second-quartile, third-quartile, and fourth-quartile) that correspond to the quartile within the storm event when the bulk of the rainfall occurs. Storms with durations of 6 hours or less, 6 to 12 hours, 12 to 24 hours, and greater than 24 hours tend to be associated with the first-, second-, third-, and fourth-quartile storms, respectively (Huff and Angel 1992).

Table RO-3
Huff Distribution for Drainage Areas from 0 to 10 Square Miles

Cumulative Storm Time (%)	Cumulative Storm Rainfall (%) for Given Storm Type			
	First Quartile (Duration ≤ 6 hours)	Second Quartile (6 < Duration ≤ 12 hours)	Third Quartile (12 < Duration ≤ 24 hours)	Fourth Quartile (Duration > 24 hours)
0	0	0	0	0
5	16	3	3	2
10	33	8	6	5
15	43	12	9	8
20	52	16	12	10
25	60	22	15	13
30	66	29	19	16
35	71	39	23	19
40	75	51	27	22
45	79	62	32	25
50	82	70	38	28
55	84	76	45	32
60	86	81	57	35
65	88	85	70	39
70	90	88	79	45
75	92	91	85	51
80	94	93	89	59
85	96	95	92	72
90	97	97	95	84
95	98	98	97	92
100	100	100	100	100

2.3 Rainfall Duration

The rainfall duration selected for any of the acceptable methods should correspond to the maximum peak flow rate for the watershed being analyzed. For the Rational Method, the duration used for determining rainfall intensity from intensity-duration-frequency data in Table RO-2 should be equivalent to the time of concentration for the watershed, calculated as outlined below in Section 3.3. For the Kinematic Wave or SCS Hydrograph Methods, a critical duration analysis should be performed to determine the duration that maximizes peak runoff rates for the watershed being analyzed. A critical duration analysis should involve applying the hydrograph-based methods to events with durations ranging from 30 minutes to 24 hours to determine the duration that produces the largest peak runoff rate for the watershed.

Guidelines for the minimum recommended storm duration based on watershed size are shown in Table RO-4. The guidelines are intended to preclude the use of short duration events that typically do not cover the corresponding watershed uniformly. The information in Table RO-4 should be considered as guidance for the minimum storm duration to use when calculating runoff and is not a replacement for a critical duration analysis.

Table RO-4
Guidelines for Minimum Storm Duration Based on Watershed Size

Watershed Size	Minimum Recommended Duration
< 160 acres	30 min
160 acres – 1 sq. mi.	1 hr
1 sq. mi. – 4 sq. mi.	2 hr
4 sq. mi. – 8 sq. mi.	3 hr
8 sq. mi. – 16 sq. mi.	6 hr
16 sq. mi. – 32 sq. mi.	12 hr
> 32 sq. mi	24 hr

3.0 RATIONAL METHOD

For urban watersheds of less than 100 acres that are not complex and do not have significant storage areas, it is acceptable to use the Rational Method to determine peak flow rates only. Due to its simplicity and inherent assumptions, it may not be appropriate for some applications. Different components of the Rational Method are explained in Sections 3.1 through 3.4. An example application of the Rational Method is presented in Section 7.1

3.1 Rational Formula

The Rational Method is based on the Rational Formula:

$$Q = CiA \qquad \text{(Equation RO-1)}$$

In which:

Q = the maximum rate of runoff cubic feet per second (cfs)

C = runoff coefficient representing the fraction of rainfall that becomes runoff

i = rainfall intensity for a duration equal to the time of concentration (in/hr)

A = drainage area (acres)

The maximum rate of runoff, Q, has units of inches per hour times acres ([in/hr]*acre); however, since this rate of (in/hr)*acre differs from cfs by less than 1 percent, the more common units of cfs are used.

The general procedure for Rational Method calculations for a single watershed is as follows:

1. Delineate the watershed boundary and calculate its area.

2. Define and measure the flow path from the upper-most portion of the watershed to the design point.

3. Calculate the slope for the flow path.

4. Calculate time of concentration, t_c (see Section 3.3).

5. Find the rainfall intensity, i, for the design storm using the calculated t_c as the duration in Table RO-1.

6. Determine the runoff coefficient, C (see Section 3.4).

7. Calculate the peak flow rate from the watershed using Equation RO-1.

3.2 Assumptions

The basic assumptions made when the Rational Method is applied are:

1. The computed maximum rate of runoff to the design point is a function of the average rainfall rate for a duration equal to the time of concentration over the drainage area.

2. The depth of rainfall used is one that occurs from the start of the storm to the time of concentration. It has a level distribution over the duration of the rainfall, meaning the rainfall intensity is constant throughout the storm.

3. The maximum runoff rate occurs when the entire area is contributing flow. However, this assumption often should be modified when a more intensely developed portion of the watershed with a shorter time of concentration produces a higher rate of maximum runoff than the entire watershed with a longer time of concentration.

3.3 Time of Concentration

The time of concentration, t_c, is defined as the time required for water to travel from the most hydraulically remote point in a watershed to the point of interest. When using the Rational Method, the rainfall duration used to determine intensity should typically equal the time of concentration of the drainage area. If a highly developed portion of the watershed produces a higher rate of runoff than the overall drainage area, then calculations should be based upon the flow length and path that results in a t_c for the highly developed portion of the drainage area only (see example in Section 7.2 of this Chapter).

For urban areas, the time of concentration, t_c, is typically calculated by breaking the flow path into reaches of overland flow, t_o, and travel time, t_t, where t_t is typically the travel time in the storm system, paved gutter or drainage channel. For non-urban areas, t_c also consists of overland flow and time of travel components where t_o is typically much longer and t_t is the time of travel in natural swales and waterways. Equation RO-2 represents the time of concentration for both urban and non-urban areas:

$$t_c = t_o + t_t \qquad \text{(Equation RO-2)}$$

In which:

t_c = time of concentration (minutes)

t_o = overland flow time (minutes)

t_t = travel time (minutes)

A minimum t_c of 10 and 5 minutes should be used for undeveloped and developed areas, respectively. Methods for calculating t_o and t_t are presented in Sections 3.3.1 and 3.3.2, respectively.

3.3.1 Overland Flow Time

The Kerby-Hathaway equation for determining the overland flow time is:

$$t_o = 0.83\left(\frac{N_k L}{S^{0.5}}\right)^{0.47} \qquad \text{(Equation RO-3)}$$

In which:

N_k = coefficient of roughness, presented in Table RO-5

L = overland flow length (ft), maximum of 500 feet for undeveloped areas and 300 feet for developed areas

S = average overland slope (ft/ft)

Table RO-5
Values of N_k for the Kerby-Hathaway Equation

Surface Type	N_k
Smooth impervious surface	0.05
Smooth bare packed soil, free of stones	0.10
Poor grass, cultivated row crops, or moderately-rough bare surfaces	0.20
Pasture or average grass cover	0.40
Deciduous timberland	0.60
Conifer timberland, deciduous timberland with deep forest litter or dense grass cover	0.80

Conservative N_k values should be chosen to provide a safety factor for adequate design. When determining predevelopment flows, a longer t_c (larger N_k value) will result in conservative design. When determining post-development flows, a shorter t_c (smaller N_k value) will result in a more conservative design. Other methods of calculating overland flow time may be acceptable pending City approval.

3.3.2 Travel Time

For watersheds with overland and channelized flow, the time of concentration must be calculated including travel time, t_t, which is calculated using the hydraulic properties of the conveyance system. The Kirpich equation for calculation of the travel time is:

$$t_t = 0.0078\left(\frac{L}{S^{0.5}}\right)^{0.77} \quad \text{(Equation RO-4)}$$

The Kirpich equation is most applicable for undeveloped watersheds with well-defined channels, bare-earth overland flow, or flow in mowed channels. The following adjustment factors are recommended for other conditions (Chow et al. 1988):

- For flow in natural grassed channels, multiply by 2.
- For overland flow on concrete or asphalt surfaces, multiply by 0.4.
- For concrete channels, multiply by 0.2.

3.3.3 Common Error in Calculating Time of Concentration

A common error when calculating time of concentration, t_c, in partly urbanized drainage areas is to neglect checking the runoff peak resulting from only the developed part of the drainage area. This check is necessary because the runoff peak from only a lower portion of the drainage area or a highly impervious area may be larger than the runoff peak from the entire drainage area. When this condition exists, the drainage area should be broken into smaller, homogeneous sub-areas to calculate the critical t_c.

3.4 Runoff Coefficient

The runoff coefficient, C, represents the percentage of rainfall that becomes runoff. The determination of C requires judgment and understanding on the part of the engineer and consideration of multiple hydrologic processes. Runoff coefficients for several land uses are provided in Table RO-6. In a non-homogeneous drainage area, C should be calculated as an area-weighted composite of the different land uses in the watershed.

Table RO-6 contains ranges of runoff coefficient values. The high end of the range shall be used when conservatively high runoff numbers are desirable. The low end of the range shall be used when conservatively low runoff numbers are desirable. Adjustments may be appropriate based on the hydrologic soil group.

The values in Table RO-6 are typical for storms with recurrence intervals of 2 to 10 years. They must be adjusted upward for less frequent recurrence intervals due to saturated soil conditions that typically occur during larger storms. Table RO-7 contains correction factors for the 25-, 50-, and 100-year events. To determine the appropriate runoff coefficient for these events, the runoff coefficient from Table RO-6 should be multiplied by the appropriate factor in Table RO-7.

Table RO-6
Runoff Coefficients Based on Surface Type for Rational Equation

By Surface Type—Use as Basis for Computation of Composite Runoff Coefficients			
Surface Type	**Runoff Coefficients**		
Asphalt, concrete pavement, roofs	0.95-1.0		
Gravel surfaces, compacted	0.85-0.95		
Gravel surfaces, not compacted	0.50-0.70		
Parks, golf courses, farms	0.10-0.20		
Lawns, pastures, hayfields			
Flat (<2% slopes)	0.10-0.15		
Average (2-7% slopes)	0.15-0.20		
Steep (>7% slopes)	0.20-0.30		
Woods	0.05-0.15		
Composite Coefficients for Single Family Residential Areas			
	Flat (<2% slopes)	Average (2-7% slopes)	Steep (>7% slopes)
Average lot size, 1/4 acre	0.35-0.45	0.40-0.50	0.45-0.55
Average lot size, 1/3 acre	0.30-0.40	0.33-0.43	0.40-0.50
Average lot size, 1/2 acre	0.25-0.35	0.30-0.40	0.36-0.46
Average lot size, 1 acre	0.20-0.25	0.25-0.30	0.30-0.38
Average lot size, 3 acres	0.10-0.20	0.16-0.24	0.25-0.33

Note: The ranges of *C* values presented in this table are typical for return periods of 2 to 10 years and assume average antecedent moisture conditions. Higher values are appropriate for larger design storms.

Table RO-7
Frequency Factors for the Runoff Coefficient
(Debo and Reese 2002)

Recurrence Interval (years)	Adjustment Multiplier
25	1.1
50	1.2
100	1.25

4.0 HYDROGRAPH METHODS

Hydrograph-based methods shall be used to calculate runoff for watersheds greater than 100 acres and smaller watersheds with complex hydrology, such as basins with land uses that are significantly non-homogeneous, or watersheds with alterations that affect runoff timing, such as detention basins and railroad/highway culverts. Hydrograph-based methods have three primary components:

1. Loss model to determine excess precipitation (runoff): The SCS Curve Number Method is an acceptable loss model (Section 4.1).

2. Hydrograph transformation model to determine the shape of the hydrograph: Acceptable hydrograph transformation models include the Kinematic Wave Method and the SCS Unit Hydrograph Method (Sections 4.3 and 4.4, respectively). The Kinematic Wave Method is the preferred approach for complex urban areas, while the SCS Method is more applicable to less developed areas.

3. Channel routing model to lag and attenuate the hydrograph as it moves downstream: The Kinematic Wave Channel Routing Method is an acceptable method for hydrograph routing in channels (Section 4.5), whereas the Modified Puls Method is acceptable for detention pond routing (Chapter 9, Detention for Flood Control).

Due to the large number of computations involved in runoff calculations and routing, computer models such as HEC-1 or its graphical successor HEC-HMS are recommended. Versions of TR-20, TR-55 or other proprietary software, which allow user input of rainfall distributions and perform acceptable detention and channel routing routines, are also acceptable. The HEC-1, HEC-HMS, TR-55 and TR-20 models are available free of charge from the agencies that developed them. These programs can be found at the following web addresses:

- HEC-1 and HEC-HMS Download from USACE: http://www.hec.usace.army.mil/

- TR-20 and TR-55 Download from NRCS: http://www.wcc.nrcs.usda.gov/

4.1 SCS Curve Number Loss Model

Sections 4.1.1 through 4.1.4 discuss key components of the SCS Curve Number Loss Model. The model requires these watershed characteristics: model area; flow path lengths, slopes and flow path characteristics (i.e., overland, grassed channel, gutter); and land use types throughout the watershed (i.e., business, residential, agricultural). The watershed boundary and area should be determined from the most current and accurate topographic maps. A field investigation should be conducted to confirm watershed characteristics. For additional information, see the documentation for HEC-1, HEC-HMS, TR-55, or TR-20.

4.1.1 SCS Curve Number Runoff Equations

The SCS Curve Number method relates a calculated Runoff Curve Number (CN) to runoff, accounting for initial abstraction losses and infiltration rates of soils. The fundamental rainfall-runoff equations are as follows:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad \text{(Equation RO-5)}$$

In which:

Q = runoff (in)

P = precipitation (maximum potential runoff) (in)

S = potential maximum watershed retention (in)

I_a = Initial abstraction (in)

Initial abstraction (I_a) is all losses before runoff begins. It includes water retained in surface depressions, water intercepted by vegetation, evaporation, and infiltration. I_a is highly variable but generally is correlated with soil and cover parameters. Through studies of many small agricultural watersheds, I_a is approximated by the following empirical equation:

$$I_a = 0.2S \qquad \text{(Equation RO-6)}$$

By removing I_a as an independent parameter, this approximation allows use of a combination of S and P to produce a unique runoff amount. Substituting Equation RO-6 into Equation RO-5 gives:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad \text{(Equation RO-7)}$$

S is related to the soil and cover conditions of the watershed through the CN. CN has a range of 0 to 100, and *S* is related to CN by:

$$CN = \frac{1000}{(S + 10)} \quad \text{(Equation RO-8)}$$

For a given CN and precipitation depth, the volume of runoff (in inches) can be calculated using Equations RO-7 and RO-8.

4.1.2 Runoff Curve Number Determination

The determination of the CN value for a watershed is a function of soil characteristics, hydrologic condition and cover or land use. CN values for undeveloped and developed areas are provided in Tables RO-8 and RO-9, respectively. For watersheds with multiple soil types or land uses, an area-weighted CN should be calculated. When significant differences in land use or natural control points exist, the watershed should be broken into smaller drainage areas for modeling purposes.

Table RO-8
Runoff Curve Number (CN) Values for Undeveloped Lands
(USDA NRCS 1986)

Cover Description	**Curve numbers for hydrologic soil group**			
Cover type and hydrologic condition	**A**	**B**	**C**	**D**
Idle lands (not yet developed)				
Pasture, grassland or range—continuous forage for grazing: Good condition (ground cover > 75% and only occasionally grazed)	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay	30	58	71	78
Woods-grass (50%-50%) combination, orchard or tree farm Other combinations can be calculated as composite of pasture and woods Good condition	32	58	72	79
Woods Good condition (i.e., woods are protected from grazing, and litter and brush adequately cover the soil)	30	55	70	77
Farmsteads—buildings, lanes, driveways and surrounding lots	59	74	82	86

Notes:

1. CN for use with SCS Unit Hydrograph Method for average runoff conditions (initial abstractions = 0.2 x Maximum Runoff Retention) (USDA NRCS 1986).
2. Typical cover condition in Springfield area is “good.” “Fair” or “poor” condition must be demonstrated by engineer prior to City approval of associated CN adjustments.
3. This table is based on average antecedent soil moisture conditions. See Section 4.1.4 for further discussion.

Table RO-9
Runoff Curve Number (CN) Values for Fully Developed and Developing Urban Areas (USDA NRCS 1986)

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	**Average percent impervious area**	**A**	**B**	**C**	**D**
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.):					
Good condition (grass cover > 75%)		39	61	74	80
Fair condition (grass cover 50% to 75%)		49	69	79	84
Poor condition (grass cover less then 50%)		68	79	86	89
Impervious areas:					
Paved parking lots, roofs, driveways, compacted gravel, etc. (excluding right-of-way)		98	98	98	98
Small open spaces within developments or ROW:		72	82	87	89
Streets and roads:					
Paved; curbs and storm sewers (including right-of-way)		90	93	95	97
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (townhouses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas: Newly graded areas (pervious areas only, no vegetation)		77	86	91	94

Notes:
1. CN for use with SCS Unit Hydrograph Method for average runoff conditions (initial abstractions = 0.2 x Maximum Runoff Retention) (USDA NRCS 1986).
2. Typical cover condition in Springfield area is “good.” “Fair” or “poor” condition must be demonstrated by engineer prior to City approval of associated CN adjustments.
3. This table is based on average antecedent soil moisture conditions. See Section 4.1.4 for further discussion.
4. Curve numbers provided for streets and roads are typical for residential or collector streets. Curve numbers for arterials and heavily developed areas should be calculated.
5. Curve numbers provided for urban districts are a typical composite of large areas. Curve numbers for individual sites should be calculated based on the proposed development.

Soil types are found in the *Soil Survey of Greene and Lawrence Counties, Missouri* (USDA NRCS 1982). Soils are classified into hydrologic soil groups (HSGs) as an indicator of infiltration rate. The HSGs are A, B, C, and D, with A having the highest infiltration rate and D having the lowest, as defined in the USDA Manual, *Urban Hydrology for Small Watersheds*, *Technical Release 55* (TR-55) (USDA NRCS 1986). The HSG and land cover are used in determining the CN value. See Table RO-10 for a summary of soils and associated HSGs typically found in the Springfield area.

For areas where the soil profile has been disturbed, the HSG should be adjusted up one level (i.e., from A to B, B to C, or C to D) unless it can be shown that the predevelopment soil profile has been reestablished.

Table RO-10
Hydrologic Soil Group (HSG) for Soils in Springfield

Soil Map Symbol	Soil Name	HSG
1B	Newtonia silt loam	B
2B	Pembroke silt loam	B
3D	Eldon cherty silt loam	B
5C	Wilderness cherty silt loam	C
6B	Creldon silt loam	C
9B	Needleye silt loam	C
10	Bado silt loam	D
11B	Sampsel silty clay loam	D
16B	Barco fine sandy loam	B
21B	Peridge silt loam	B
23B	Bolivar fine sandy loam	B
24	Parsons silt loam	D
26D	Collinsville fine sandy loam	C
27D	Basehor stony fine sandy loam	D
30C	Keeno cherty silt loam	C
32C	Freeburg & Alsup silt loams	C
33B	Keeno & Eldon cherty silt loams	C
35D	Clarksville & Nixa cherty silt loams	B
40E	Alsup very stony silt loam	C
43D	Goss cherty silt loam	B
44E	Goss-Gasconde complex	B
45E	Clarksville cherty silt loam	B
50C	Nixa cherty silt loam	C
53B	Wilderness & Goss cherty silt loams	C
54	Lanton silt loam	D
55	Huntington silt loam	B
56	Osage silty silt loam	D
61B	Hoberg silt loam	C
76	Hepler silt loam	D
81B	Viraton silt loam	C
83D	Gasconade-Rock Outcrop complex	D
94	Cedargap cherty silt loam	B
95	Cedargap silt loam	B
240	Gerald silt loam	D
241B	Parsons & Sampsel silt loams	D
245	Carytown silt loam	D
921	Secesh & Cedargap silt loams	B
931	Waben & Cedar silt loams	B
940	Dumps-Orthents	C
941	Pits & Dumps	C
943	Orthents	C

Source: *Soil Survey of Greene and Lawrence Counties, Missouri* (USDA NRCS 1982)

4.1.3 Cover Types

Tables RO-8 and RO-9 address most cover types, such as vegetation, bare soil, and impervious surfaces that are commonly encountered in urban areas. A number of methods exist for determining cover type. The most common are field reconnaissance, aerial photographs, and land use maps.

4.1.4 Hydrologic Condition

Hydrologic condition indicates the effects of cover type and treatment on infiltration and runoff and is generally estimated from density of plant and residue cover on sample areas. *Good* hydrologic condition indicates that the soil usually has a low runoff potential for a specific HSG, cover type, and treatment. Some factors to consider in estimating the effect of cover on infiltration and runoff are the canopy or density of lawns, crops, or other vegetative areas, and the amount of year-round cover and seasonal cover such as deciduous leaves. Typical cover condition in the Springfield area is good. In rare cases, a lesser amount of ground cover exists in a natural condition. Fair or poor condition must be demonstrated by the engineer prior to City approval of associated CN adjustments.

4.1.5 Antecedent Moisture Conditions

The index of runoff potential before a storm event is the Antecedent Moisture Condition (AMC). The AMC accounts for the existing degree of soil saturation at the beginning of a rainfall, therefore adjusting the CN to reflect more accurate runoff conditions. All values given in Tables RO-8 and RO-9 represent AMC 2 (median moisture conditions) and should be used for design. Adjustments for AMC 1 (dry conditions) and AMC 3 (wet conditions) can be made if appropriate (see USDA NRCS 1986, TR-55).

4.2 Kinematic Wave Hydrograph Method

A hydrograph method must be used to calculate runoff for watersheds greater than 100 acres and smaller watersheds with complex hydrology. The Kinematic Wave Method is the preferred method for computing watershed runoff, particularly for urbanized watersheds. This overland flow hydrograph method is applied in conjunction with channel routing methods described in Section 4.4 to transform rainfall to runoff and route it to determine the hydrograph at the point of interest at the low point of the drainage area.

4.2.1 Fundamental Equations

To apply the Kinematic Wave Method to a typical watershed (Figure RO-1a), the watershed and its channel are conceptualized as shown in Figure RO-1b. In Figure RO-1b, the watershed is represented as two plane surfaces over which water runs until it reaches the channel. At a cross section, the system resembles an open book, with the water running parallel to the text on the page (down the shaded planes) and then into the channel that follows the book's center binding. The Kinematic Wave Method represents behavior of overland flow on the plane surfaces. The model may also be used to simulate behavior of flow in the watershed channels as discussed in Section 4.4.

The momentum and continuity equations used in the Kinematic Wave Method are solved using finite difference methods. Details of the method are beyond the scope of this manual. These calculations are best performed using a computer program such as HEC-1 or HEC-HMS. For more background information on the derivation of the Kinematic Wave Method, refer to the HEC-1 or HEC-HMS User's Manual or other reference.

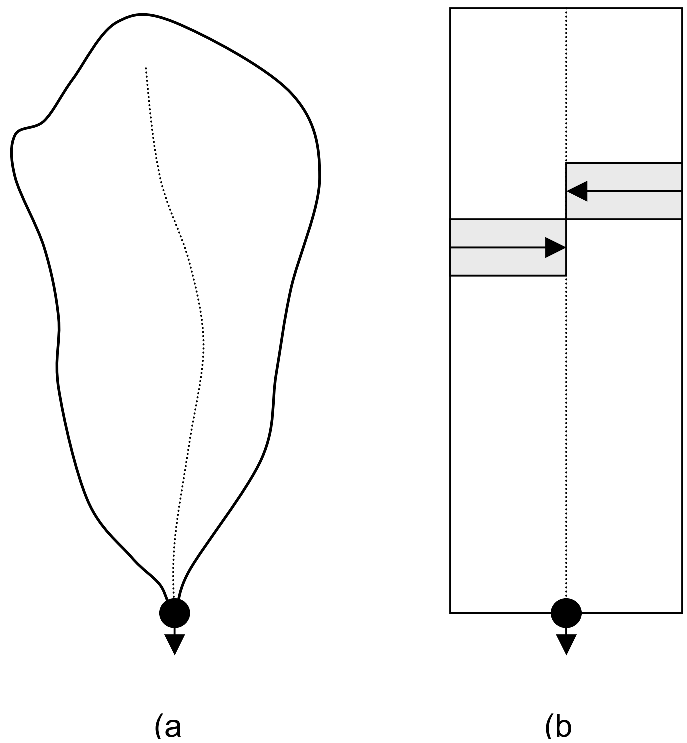

Figure RO-1
Kinematic Wave Representation of Simple Watershed

4.2.2 Kinematic Wave Model Parameters

To determine runoff using the Kinematic Wave Method, the watershed must be modeled as a set of elements including:

- **Overland Flow Planes:** This model input describes one or two planes that contribute runoff to channels within the watershed. The combined flow from the planes is the total inflow to the watershed channels. Information that must be provided about each plane is listed in the "Overland Flow Panes" column of Table RO-11. To determine runoff from rainfall on these planes, the Kinematic Wave Overland Flow Method is applied.

- **Sub-collector Channels:** These are small feeder pipes or channels that convey water from street surfaces, rooftops, lawns, etc. Sub-collector channels might service a portion of a city block or housing tract with an area of 10 acres or less. Flow is assumed to enter the channel uniformly along its length. The average contributing area for each sub-collector channel must be identified as a part of model input. Information that must be provided about the sub-collector channels is listed in the “Collectors and Sub-Collectors” column of Table RO-11. The Kinematic Wave Channel Routing Method (Section 4.4) is applied to route flows through sub-collector channels.

- **Collector Channels:** These channels collect flow from sub-collector channels and convey it to the main channel. Collector channels might service an entire city block or housing tract, with flow entering laterally along the length of the channel. Similar to the sub-collector channels, the average contributing area for each collector channel must be identified. Information that must be provided about the collector channels is listed in the “Collectors and Sub-Collectors” column of Table RO-11. The Kinematic Wave Channel Routing Method (Section 4.4) is applied to route flow through collector channels.

- **Main Channel:** The main channel conveys flow from upstream sub-watersheds and flows that enter from the collector channels or overland flow planes. Information that must be provided about the main channel is listed under the “Main Channel” heading in Table RO-11.

The choice of elements to describe any watershed depends upon the configuration of the drainage system. The minimum configuration is one overland flow plane and the main channel. A more complex system may require two planes, sub-collectors, collectors, and the main channel.

Table RO-11
Required Input for Kinematic Wave Model

Parameter	Kinematic Wave Model Element		
	Overland Flow Planes	**Collectors and Sub-collectors**	**Main Channel**
Length	Typical length	Representative channel length	Channel length
Area	Area represented by plane	Area drained by channel	-
Slope	Representative Slope	Representative channel slope	Channel slope
Roughness	Overland-flow roughness coefficient	Representative Manning’s roughness coefficient	Representative Manning’s roughness coefficient
Channel Geometry	-	Principle dimensions of representative channel cross section; Description of channel shape	Principle dimensions of channel cross section
Other	Loss model parameters (CN value)	-	Identification of upstream inflow hydrograph (if any)

See Table RO-12 for a list of roughness coefficients for overland flow. For channel roughness coefficients refer to Chapter 8, Open Channels.

Table RO-12
Roughness Coefficients for Kinematic Wave Method

Surface Description	*n*
Smooth surfaces (concrete, asphalt, gravel or bare compacted soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses*	0.24
Bermuda grass	0.41
Range	0.13
Woods:**	
Light underbrush	0.40
Dense underbrush	0.80

*Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama, and native grass mixtures.
**When selecting *n*, consider cover to a height of approximately 0.1 ft. This is the only part of plant cover that will obstruct sheet flow.

4.3 SCS Unit Hydrograph Method

The SCS Unit Hydrograph Method is one of the methods appropriate for drainage areas larger than 100 acres and complex watersheds. While it is an acceptable method for any size drainage area, it is a lumped parameter model and may not be as preferable as the Kinematic Wave Method in many cases. The determination of the storm runoff hydrograph for a specified total rainfall using the SCS Method involves three primary steps:

1. Develop the unit hydrograph for the watershed, based on the watershed characteristics and time of concentration.

2. Determine the excess precipitation values using the CN value and rainfall values.

3. Calculate the storm runoff hydrograph by applying the excess precipitation values in Step 2 to the unit hydrograph values in Step 1.

Sections 4.3.1 through 4.3.3 describe the SCS Unit Hydrograph Method in more detail.

4.3.1 Time of Concentration

The time of concentration, t_c, influences the shape and peak of the runoff hydrograph. The t_c for a watershed is determined based on overland flow time, t_o, and travel time through the watershed's hydrologic conveyance system, t_t. Procedures for calculating t_c are presented in Section 3.3 of this Chapter.

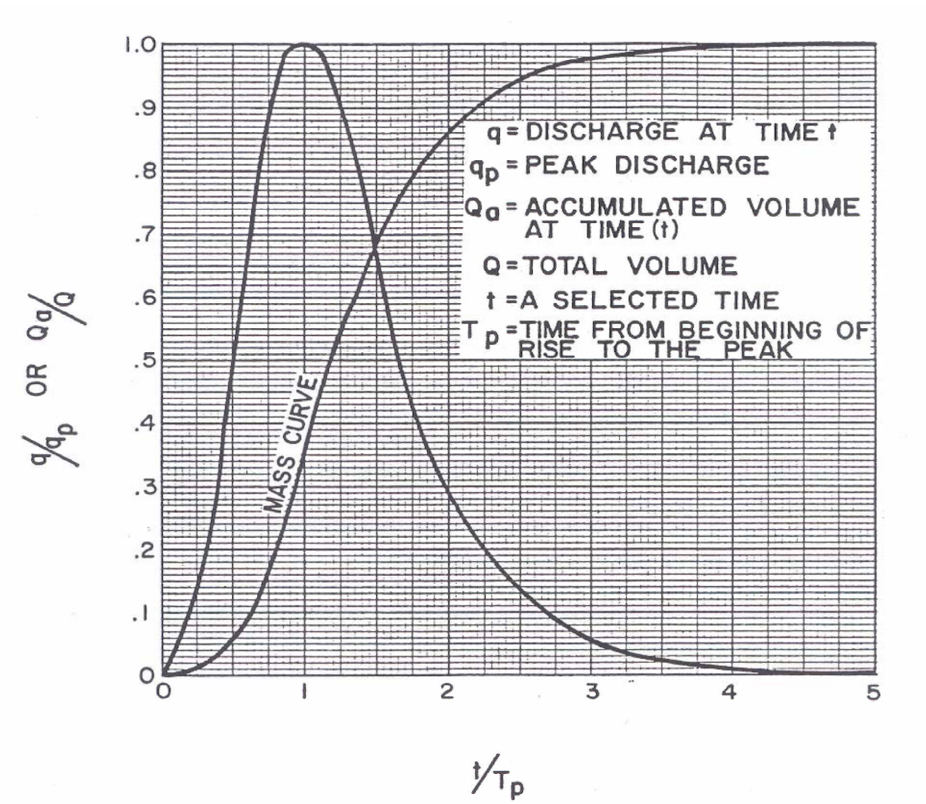

Figure RO-2
SCS Dimensionless Unit Hydrograph

4.3.2 Unit Hydrograph

A dimensionless unit hydrograph has been developed by the SCS based on the evaluation of a large number of natural unit hydrographs from various watersheds. The SCS dimensionless unit hydrograph is presented in Figure RO-2.

To determine the unit hydrograph for a specific watershed, the following relationships are used:

$$q_p = 484\frac{A}{T_p} \qquad \text{(Equation RO-9)}$$

$$T_p = \frac{\Delta D}{2} + 0.6t_c$$ (Equation RO-10)

In which:

q_p = peak discharge (cfs)

A = watershed area (square miles)

T_p = time to peak (hours)

t_c = time of concentration (hours)

ΔD = duration of excess rainfall (hours)

484 = peaking factor related to the ratio of the unit hydrograph time base to the time to peak and unit conversions to provide q_p in units of cfs.

The unit hydrograph values are obtained by multiplying q_p and T_p for the selected duration of excess rainfall, ΔD. The duration of excess rainfall should be selected so that ΔD is approximately equal to $0.133t_c$.

4.3.3 Storm Runoff Hydrograph

The storm runoff hydrograph is determined by multiplying the excess precipitation values by the unit hydrograph values and summing the ordinates in a specific manner. The method is based on the following primary assumptions: 1) discharge at any time is proportional to the volume of runoff, and 2) time factors affecting the hydrograph shape are constant. For more detailed theoretical discussion, refer to Section 4 of the *SCS National Engineering Handbook* (USDA SCS 1966). An example calculation using the SCS Unit Hydrograph Method is provided in Section 7.3.

4.4 Channel Routing of Hydrographs

For drainage areas larger than 100 acres or that are complex or non-homogeneous, sub-areas of homogeneous land use should be developed in the model. Hydrographs from each sub-area must be routed and combined to determine the hydrograph for the entire drainage area. The Kinematic Wave Channel Routing Method is the preferred method for this procedure, although other methods may be acceptable upon approval. Where appreciable hydrograph attenuation is anticipated due to storage effects along a reach, a method that explicitly accounts for channel storage effects, such as the Modified-Puls method (presented in Chapter 9, Detention for Flood Control), may also be approved.

4.4.1 Kinematic Wave Channel Routing Method

The Kinematic Wave Channel Routing Method is used to route an upstream inflow hydrograph through a reach with known geometric characteristics. Theoretically, a flood wave routed by the Kinematic Wave

Channel Routing Method is translated, but not attenuated, through a reach (although a degree of attenuation is introduced by the finite difference solution to the governing equations). The lack of significant peak attenuation during hydrograph translation is a fairly common characteristic of urban conveyances.

The theory of the Kinematic Wave Method presented in Section 4.3 for overland flow is also applicable to channel routing. This method is typically applied using software such as HEC-1 or HEC-HMS. Table RO-13 summarizes input parameters required for the Kinematic Wave Channel Routing Method. Manning's roughness values should be selected in accordance with Chapter 8, Open Channels.

Table RO-13
Kinematic Wave Channel Routing Method Inputs

Input Parameter	Note
Length (ft)	Determine as actual length of flow path along thalweg.
Slope (ft/ft)	Calculate as change in elevation divided by channel length.
Manning's *n*	Determine according to Chapter 8, Open Channels.
Shape	Trapezoid, deep or circular. Trapezoidal can also be used for rectangular and triangular cross-sections by specifying appropriate side slopes and bottom width. Use deep channel when flow depth ≈ channel width.
Width or Diameter (ft)	Characteristic dimension (bottom width for trapezoidal; channel width for deep; and diameter for circular).
Side Slope (H:V) (ft/ft)	For trapezoidal channels only.
Minimum Number of Routing Increments	The minimum number of steps is related to the finite difference solution of the governing equations. The minimum number of routing increments is automatically determined by the program but optionally can be entered by the user (not recommended by City).

4.5 Reservoir Routing of Hydrographs

For watersheds with significant detention structures, the effects of routing hydrographs through facilities can have important implications on the timing of peak flow rates from sub-watersheds. Hydrologic modeling and analysis must account for the effects of detention by performing reservoir routing calculations. The criteria and methods for reservoir routing are presented in Chapter 9, Detention for Flood Control. Options for routing using common methods are included in HEC-1, HEC-HMS, TR-20 and many other commercially available hydrology software packages.

5.0 ALTERNATIVE METHODS

5.1 USGS Regression Equations

For urban watersheds larger than 100 acres, the engineer should compare the peak flow rate determined from the critical duration analysis with the peak flow rate calculated using the USGS regression equations in *The National Flood-Frequency Program—Methods for Estimating Flood Magnitude and Frequency in*

Rural and Urban Areas in Missouri, 2000 (Knowles and Mason 2000). These equations are summarized in Table RO-14. Guidance for determining the basin development factor (BDF) is provided by Knowles and Mason (2000). The set of equations including impervious area should be used only in watersheds with a very low amount of development or where information required for determining the BDF is not available. These equations do not apply when there is significant storage in the watershed. Although the resulting peak flow rates typically have a relatively high coefficient of variation, regression equations provide a useful reasonableness check for urban watersheds.

Table RO-14
USGS Regression Equations for Estimation of Peak Flow in Urban Areas (Knowles and Mason 2000)

Recurrence Interval	USGS Regression Equation (Q = estimated peak flow rate [cfs], A = drainage area [mi^2], BDF = basin development factor, I = impervious area in percent)
Regression Equations Based on Basin Development Factor (BDF)	
2-year	$Q_2 = 801\ A^{0.747}\ (13 - BDF)^{-0.400}$
5-year	$Q_5 = 1150\ A^{0.746}\ (13 - BDF)^{-0.318}$
10-year	$Q_{10} = 1440\ A^{0.755}\ (13 - BDF)^{-0.300}$
25-year	$Q_{25} = 1920\ A^{0.764}\ (13 - BDF)^{-0.307}$
50-year	$Q_{50} = 2350\ A^{0.773}\ (13 - BDF)^{-0.319}$
100-year	$Q_{100} = 2820\ A^{0.783}\ (13 - BDF)^{-0.330}$
Regression Equations Based on Imperviousness	
2-year	$Q_2 = 224\ A^{0.793}\ I^{0.175}$
5-year	$Q_5 = 424\ A^{0.784}\ I^{0.131}$
10-year	$Q_{10} = 560\ A^{0.791}\ I^{0.124}$
25-year	$Q_{25} = 729\ A^{0.800}\ I^{0.131}$
50-year	$Q_{50} = 855\ A^{0.810}\ I^{0.137}$
100-year	$Q_{100} = 986\ A^{0.821}\ I^{0.144}$

6.0 OFFSITE AND ADJACENT RUNOFF

The design of a drainage system should take into account the runoff from offsite and adjacent areas, recognizing their urban development potential. For hydrologic analysis, calculations for offsite areas should assume these areas are completely developed, even if they are undeveloped or have not yet reached their full development potential. For the purposes of calculations and modeling, land use for offsite areas should be based on City zoning and anticipated land use for these areas. For flood control planning and modeling, effects of detention should be disregarded except for publicly owned and maintained facilities with storage dedicated for perpetuity, unless otherwise approved by the City. Fully developed flows should be used for facility sizing and design. The City may request additional modeling including the effects of detention in the watershed, especially where timing of peak flow rates is a concern. See Chapter 8, Open Channels and Chapter 9, Detention for Flood Control.

7.0 EXAMPLES

7.1 Rational Method

Find the 100-year peak flow rate for a 40-acre drainage area consisting of single-family residential land use with an average lot size of ¼ acre with good grass cover and maximum ground slope of about 2 percent. The upper 200 feet of the watershed is sloped at 2 percent, and the lower 1,100 feet is a grassed waterway sloped at 1 percent.

Determine Runoff Coefficient:

From Table RO-6, for single-family residential land use with ¼-acre lots, slope of 2 percent and 2- to 10-year rainfall, the runoff coefficient is 0.45. For a 100-year rainfall, this value must be multiplied by 1.25, resulting in a runoff coefficient of 0.56 (Table RO-7).

Determine t_c:

Calculate the overland flow segment, t_o, with average grass cover (N_k = 0.40) using the Kerby-Hathaway formula (Equation RO-3):

$$t_o = 0.83\left(\frac{0.40 \cdot 200}{0.02^{0.5}}\right)^{0.47}$$

$$t_o = 16.3 \text{ minutes}$$

Calculate the travel time segment, using the Kirpich Formula (Equation RO-4):

$$t_t = 0.0078\left(\frac{1100}{0.01^{0.5}}\right)^{0.77}$$

$$t_t = 10.1 \text{ minutes}$$

From Equation RO-2:

$$t_c = 16.3 + 10.1$$

$$t_c = 26.4 \text{ minutes}$$

The Tc and PeakQ Worksheet in the SF-Rational Spreadsheet can also be used to calculate time of concentration.

Determine Rainfall Intensity:

Determine rainfall intensity, *i*, from Table RO-2 for a 100-year return interval. Since there is not an entry in Table RO-2 for t = 26 minutes, use linear interpolation with bounding values to determine $i_{26\,min}$:

$$i_{15\min} = 8.84\ in/hr$$

$$i_{30\min} = 6.06\ in/hr$$

$$i_{26\min} = i_{30\min} + \frac{(i_{15\min} - i_{30\min})}{(30-15)}(30-26)$$

$$i_{26\min} = 6.06 + \frac{(8.84-6.06)}{(30-15)}(30-26)$$

= 6.8 in/hr

Determine Q_{100} from Equation RO-1:

$$Q = CiA = 0.56 \cdot 6.8 \cdot 40$$

Q_{100} = 152 cfs

7.2 Rational Method: Peak Flow from Portion of a Drainage Area

Find the 5-year peak flow rate at a point with a 15-acre drainage area, as shown in Figure RO-3. The lower portion of the watershed is a 12-acre parking area. The remainder of the watershed is a flat (slope < 2 percent) grassed area. Using the methods described in Section 3.3, it is determined the time of concentration for the entire watershed is 20 minutes and the time of concentration is 15 minutes for the parking lot only.

Determine Runoff Coefficients:

From Table RO-6, the appropriate runoff coefficient for a new asphalt parking lot is 1.0, and the runoff coefficient for a flat, grassed area is 0.15. A runoff coefficient for the entire watershed can be calculated by area-weighting of these runoff coefficients:

$$C_{watershed} = \frac{C_{asphalt} \cdot A_{asphalt} + C_{grass} \cdot A_{grass}}{A_{asphalt} + A_{grass}}$$

$$C_{watershed} = \frac{1.0 \cdot 12 + 0.15 \cdot 3}{12+3}$$

$$C_{watershed} = 0.83$$

The Weighted C Worksheet in SF-Rational Spreadsheet can also be used to calculate the runoff coefficient.

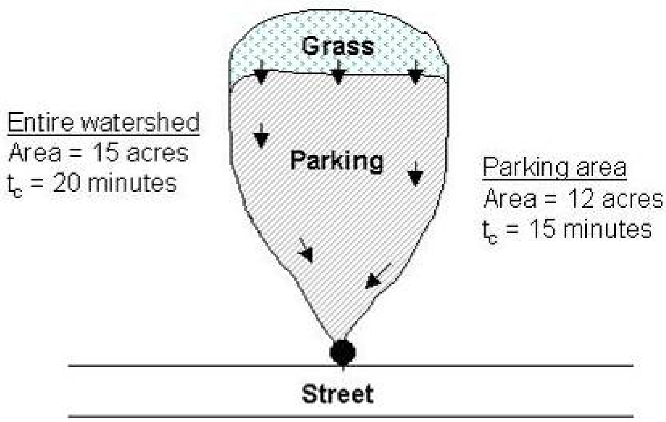

Figure RO-3
Example Watershed for Rational Method

Determine Rainfall Intensity:

From Table RO-2 for a 5-year return interval, rainfall intensities corresponding to t_c = 15 and 30 minutes are 5.16 and 3.54 inches/hour, respectively. Since there is not an entry in Table RO-2 for t = 20 minutes, use linear interpolation with bounding values to determine $i_{20\,min}$:

$$i_{15\,\min} = 5.16\ in/hr$$

$$i_{30\,\min} = 3.54\ in/hr$$

$$i_{20\,\min} = i_{30\,\min} + \frac{(i_{15\,\min} - i_{30\,\min})}{(30-15)}(30-20)$$

$$i_{20\,\min} = 3.54 + \frac{(5.16-3.54)}{(30-15)}(30-20)$$

$$= 4.62 \text{ in/hr}$$

Calculate Peak Flow Rates:

Determine Q from Equation RO-1 or by using the Tc and PeakQ Worksheet in the SF-Rational Spreadsheet.

For the parking area only:

$$Q_5 = 1.0 \cdot 5.16 \cdot 12$$

$$= 61.9 \text{ cfs}$$

For the entire watershed:

$$Q = 0.83 \cdot 4.62 \cdot 15$$

$$= 57.5 \text{ cfs}$$

The appropriate Q_5 to use for the design point is 61.9 cfs. Because of the shorter time of concentration (higher design rainfall intensity) and the higher runoff coefficient, the parking area alone will produce a higher peak runoff rate than the drainage area considered as a whole.

7.3 SCS Unit Hydrograph Method

Determine the 100-year, 2-hour peak flow rate for a watershed with the following characteristics:

Area = 1.2 square miles (767 acres)

HSG = C

Land uses:

1.1. Residential (¼-acre lots)—267 acres

1.2. Residential (½-acre lots)—300 acres

1.3. Commercial—100 acres

1.4. Park/open space (good)—100 acres

From methods given in Section 3.3,

t_c = 45 minutes

<u>Determination of Unit Hydrograph:</u>

Select time interval such that:

$$\Delta D = 0.133 t_c$$

= 0.133 (45) = 5.99 minutes; use 6 minutes

From Equation RO-10:

$$T_p = \frac{6}{2} + 0.6(45)$$

= 30 minutes = 0.5 hours

From Equation RO-9:

$$q_p = 484\left(\frac{1.2}{0.5}\right)$$

= 1,162 cfs

Reading q/q_p and t/t_p values from Figure RO-2 and multiplying by the computed q_p value, develop the unit hydrograph shown in Table RO-15.

Table RO-15
Example Unit Hydrograph Tabulation

t (min)	t/T_p	q/q_p	q (cfs)
(1)	(2)	(3)	(4)
0	0.00	0.00	0
6	0.20	0.1	116
12	0.40	0.37	430
18	0.60	0.63	732
24	0.80	0.91	1057
30	1.00	1.00	1162
36	1.20	0.94	1092
42	1.40	0.88	1022
48	1.60	0.53	616
54	1.80	0.39	453
60	2.00	0.285	331
66	2.20	0.21	244
72	2.40	0.155	180
78	2.60	0.11	128
84	2.80	0.08	93
90	3.00	0.055	64
96	3.20	0.004	5
102	3.40	0.0025	3
108	3.60	0.002	2
114	3.80	0.0015	2
120	4.00	0.001	1

Column (1) Time elapsed in increments of ΔD (ΔD = 6 minutes, approximately equal to $0.133t_c$)
Column (2) = Col (1) / T_p (T_p = 30 minutes, calculated in example using equation RO-10)
Column (3) = Read from Figure RO-2 based on t/t_p value in Col (2)
Column (4) = q_p*Col (3) (q_p is calculated in example using equation RO-9).

Calculate Excess Precipitation:

Calculate composite CN for watershed using Table RO-10 and HSG C:

Residential (¼ acre)—267 acres: CN = 83

Residential (½ acre)—300 acres: CN = 80

Commercial—100 acres: CN = 94

Park/open space (good)—100 acres: CN = 74

The composite CN can be calculated based on area weighting as follows:

$$CN_{composite} = \frac{(267 \cdot 83) + (300 \cdot 80) + (100 \cdot 94) + (100 \cdot 74)}{(267 + 300 + 100 + 100)}$$

= 82

Determine rainfall total and apply appropriate distribution:

From Table RO-1, the total rainfall for a 100-year, 2-hour event is 4.74 inches. A Huff first quartile distribution should be used (Table RO-3) to develop Table RO-16.

Table RO-16
Example Precipitation Distribution

Cumulative Storm Time (%)	Cumulative Precipitation (%)	Storm Time (min)	Cumulative Precipitation (in)
(1)	(2)	(3)	(4)
0	0	0	0.00
5	16	6	0.76
10	33	12	1.56
15	43	18	2.04
20	52	24	2.46
25	60	30	2.84
30	66	36	3.13
35	71	42	3.37
40	75	48	3.56
45	79	54	3.74
50	82	60	3.89
55	84	66	3.98
60	86	72	4.08
65	88	78	4.17
70	90	84	4.27
75	92	90	4.36
80	94	96	4.46
85	96	102	4.55
90	97	108	4.60
95	98	114	4.65
100	100	120	4.74

Column (1) From Table RO-3
Column (2) From Table RO-3, First Quartile
Column (3) = storm time; ΔD = 6 minutes per time step

Column (4) = 4.74*Col (2) (4.74 value from Table RO-1 for 100-year, 2-hour event)

Calculate S based on the CN value (using Equation RO-8):

$$82 = \frac{1000}{S+10}$$

$$S = \frac{1000}{82} - 10$$

$$= 2.19 \text{ in}$$

Using the tabulated precipitation values in Table RO-16, determine excess precipitation for each time period using Equation RO-7. Excess precipitation results for each time period are tabulated in Table RO-17.

Table RO-17
Example Excess Precipitation Tabulation

Storm Time (min)	Cumulative Precipitation (in)	Accumulated Runoff (in)	Incremental Runoff (in)
(1)	(2)	(3)	(4)
0	0.00	0.00	0.00
6	0.76	0.04	0.04
12	1.56	0.38	0.34
18	2.04	0.68	0.29
24	2.46	0.97	0.30
30	2.84	1.26	0.29
36	3.13	1.48	0.22
42	3.37	1.67	0.19
48	3.56	1.83	0.16
54	3.74	1.99	0.16
60	3.89	2.11	0.12
66	3.98	2.19	0.08
72	4.08	2.27	0.08
78	4.17	2.35	0.08
84	4.27	2.43	0.08
90	4.36	2.52	0.08
96	4.46	2.60	0.08
102	4.55	2.68	0.08
108	4.60	2.73	0.04
114	4.65	2.77	0.04
120	4.74	2.85	0.08

Column (1) = storm time; ΔD = 6 minutes per time step
Column (2 = from Table RO-16, column 4.
Column (3) =((Col (2)-0.2*S)^2)/(Col (2) + 0.8*S) (S = 2.19, calculated using Equation RO-10).
Note if value in Col (2) is less than 0.2*2.19, Col (3) = 0
Column (4) =Col (3) Row (i) - Col (3) Row (i-1)

Calculate Runoff Hydrograph:

The runoff hydrograph is calculated by multiplying the ordinates of the unit hydrograph by the excess precipitation for each time increment. This applies for all time increments in which there is excess precipitation to derive a hydrograph. These incremental hydrographs are lagged so the start of the incremental hydrograph corresponds to the time of the excess precipitation it represents. Incremental hydrographs are superimposed to derive the runoff hydrograph (Table RO-18).

Due to the extensive amount of time required to complete these computations by hand, an SCS Unit Hydrograph Method computation is typically completed through the aid of computer software such as TR-55, TR-20, HEC-1, or HEC-HMS.

Table RO-18
Example Runoff Hydrograph Tabulation

Time (min)	Unit hydrograph (cfs)	Excess Precip (in) / Time (min)									Storm Hydrograph (cfs)
		0.041	0.34	0.29	0.3	0.29	0.22	0.19	0.16	…	
		6	12	18	24	30	36	42	48	…	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
0	0	0									0
6	116	5	0								5
12	430	18	39	0							57
18	732	30	146	34	0						210
24	1057	43	249	125	35	0					452
30	1162	48	359	212	129	34	0				782
36	1092	45	395	307	220	125	26	0			1116
42	1022	42	371	337	317	212	95	22	0		1396
48	616	25	347	317	349	307	161	82	19	0	1606
54	453	19	209	296	328	337	233	139	69	…	1629
60	331	14	154	179	307	317	256	201	117	…	1543
66	244	10	113	131	185	296	240	221	169	…	1365
72	180	7	83	96	136	179	225	207	186	…	1119
78	128	5	61	71	99	131	136	194	175	…	872
84	93	4	44	52	73	96	100	117	164	…	649
90	64	3	32	37	54	71	73	86	99	…	454
96	5	0	22	27	38	52	54	63	72	…	329
102	3	0	2	19	28	37	40	46	53	…	224
108	2	0	1	1	19	27	28	34	39	…	150
114	2	0	1	1	2	19	20	24	29	…	95
120	1	0	1	1	1	1	14	18	20	…	56

Column (1) = storm time; ΔD = 6 minutes per time step
Column (2 = calculated q (column 4 from Table RO-15)
Column (3) through (11) = Excess precipitation (top value), from column 4 of Table RO-18, for each time step (ΔD).
Column (12) = sum of columns (3

7.4 Kinematic Wave Method

This example demonstrates conceptual model development and model input requirements for the Kinematic Wave Hydrograph and Channel Routing Methods for analysis of runoff for a proposed residential development. The preferred computer programs to perform the computations are HEC-1 or HEC-HMS. The 20-acre development will consist of ⅓-acre residential lots constructed in an area with HSG C soils. Figure RO-4 illustrates the example watershed and important input parameters.

Model Conceptualization:

Development of the Kinematic Wave hydrograph model requires the determination of the number of overland flow planes, main channels, collector channels and sub-collector channels. For accurate modeling of urban areas, it is generally advisable to use two overland flow planes, one for pervious area in the watershed, the other for impervious area. This provides a more realistic model of rainfall-runoff from impervious areas than a model using a single overland flow plane that lumps pervious and impervious areas together. The overland flow plane conceptualization is illustrated in Figure RO-4.

It is also necessary to model a realistic flow network. For smaller drainage areas, a network consisting of a main channel and collector channels of representative cross section, slope, length, and roughness will typically suffice. For larger watersheds, sub-collector channels, which feed into collector channels, should also be used. For this 20-acre drainage area, sub-collector channels are not necessary, so a network consisting of a main channel and collector channels will be used.

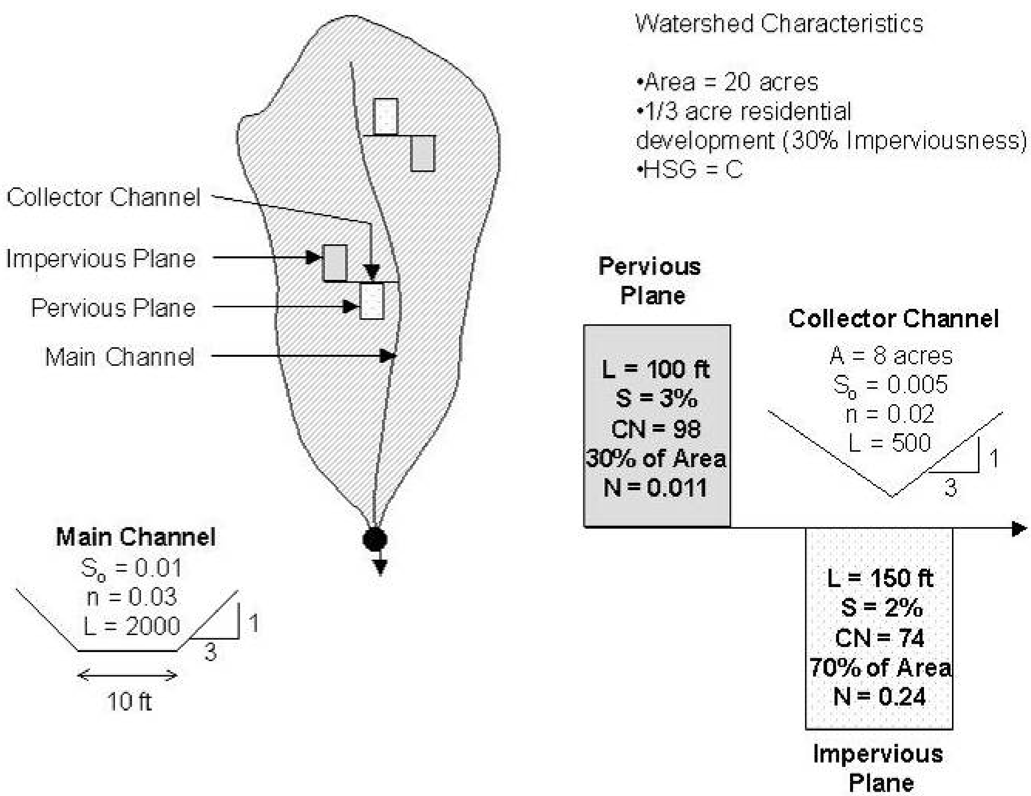

Figure RO-4
Example Watershed for Kinematic Wave Method

Overland Flow Planes:

For the overland flow planes, it is necessary to provide CN values. These can be selected for each plane from Table RO-9. For impervious area, a CN of 98 is appropriate; for pervious areas in a residential subdivision with restored soil profiles (HSG C matches predevelopment HSG C) the appropriate CN is 74. The percent of the total drainage area associated with each plane should be determined from development plans. For preliminary calculations, the guidance in Table RO-11 can be used. In this case, with a 1/3-acre residential development, 30 percent of the area is assigned to the impervious plane, with the remainder assigned to the pervious plane.

Overland flow path length, slopes, and *n* values must be provided for each plane. Representative flow path lengths and slopes can be determined from the construction and grading plans of the development. Figure RO-4 quantifies these parameters for the example watershed. Table RO-12 can be used to determine *n* values. A value of n = 0.011 (smooth surfaces) is used for the impervious plane; a value of n = 0.24 (dense grass) is used for the pervious plane.

Channels:

Channel characteristics must be specified for collector channels and the main channel. Required inputs for channels are summarized in Table RO-11 and include representative cross-sectional geometry, flow path length, channel slope and Manning's roughness coefficient, *n*, for open channel flow. For each collector channel, the area tributary to the collector channel must be specified. Geometric parameters are determined from proposed construction and grading plans. Manning's *n* for open channels should be selected from Chapter 8, Open Channels, not from Table RO-12 in this Chapter.

Required channel routing inputs for the Kinematic Wave model are presented on Figure RO-4.

8.0 REFERENCES

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